

# Structural Pressure Values of Natural and Chemically Treated Soils

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## Abstract

Deformations of soils dramatically increase when applied loads exceed a certain load limit which is defined as soil structural pressure. Structural pressure values may be used as identifiers of boundaries of active zones below foundations which separate “elastic-plastic” zones of base soils from the ones which are in the “elastic” state. The paper presents a method for calculation of structural pressure values using oedometer test results. Simple formulas are suggested for calculations of structural pressure values which are unique properties of each natural and chemically treated soil. The rule for the selection of experimental input information that may be used in the calculation procedure is explained. The paper includes examples illustrating the proposed standardized calculation procedure of structural pressure values using the published results of one-dimensional compression tests performed on different soils or man-made materials.

## Keywords

Oedometer; Test Results; Structural Pressure; Values

## Introduction

The difference between the stress-strain curves which were obtained in oedometer tests performed on undisturbed and disturbed soil specimens (which were recovered from the same depths and represented “identical” soils having the same moisture contents) brought to the light the necessity to study structural characteristics of soils. Pre-consolidation pressure values,  $P_c$ , which were introduced by Casagrande (1932, 1936), were accepted by geotechnical engineers and scientists as a logical explanation of soil sedimentation process. However, it was recognized by many workers (Schmertmann, 1955; Crawford, 1986; Leroueil and Vaughan, 1990; Wesley, 1990; etc) that further research is needed to better understand the influence of soil structures on soil mechanical properties. Leonards (1962) wrote that “interparticle forces can be altered by a variety of factors other than pressures” (Leonards, 1962, p. 150). Fig. 2-27 of Leonard’s work (Leonards, 1962, p. 151) contains the results of three oedometer

tests performed by Girault (1960) on undisturbed specimens of Mexico City clay recovered from the same depth. The pore water in one of specimens was substituted with carbon tetrachloride. As a result, the compressibility of that specimen significantly differed from the compressibility of other two specimens (where pore water was not changed). The pre-consolidation stress value,  $P_c$ , also changed meaning that pre-consolidation pressure value,  $P_c$ , did not depend only on the weight of soils accumulated in the past at the area of interest.

Working with residual soils, Vargas (1973) noted that oedometer curves indicate the existence of “specific” points corresponding to strain rate increases observed during oedometer testing of sedimentary soils. It was shown by Wesley that the definition of pre-consolidation pressure,  $P_c$ , identifying the “connection between behavior and stress history applicable to sedimentary soils is not relevant to residual soils” (Wesley, 1990). Soil structural pressure values,  $\sigma_{sz}$ , are the maximum stress values for which only recoverable deformations may be observed during the loading/unloading process ( $z$  is the depth of a soil point below the loaded area). It is possible to calculate  $\sigma_{sz}$  using the results of oedometer tests. This paper presents a calculation procedure and its application to determination of structural pressure values using the results of oedometer tests, performed on natural soils of different origin as well as chemically improved ones. The examples illustrating step-by-step calculation technique are included in the paper.

## Proposed Formulas and Illustrative Examples

Any oedometer curve may be described in the following way (Figure 1)

$$(\Delta h_g/h_0)_{total} = \delta + r \quad (1)$$

or

$$(\Delta h_g/h_0)_{total} = (e_0' - e_g)/(1 + e_0) \quad (1a)$$

where  $\delta = (e_0' - e_0)/(1 + e_0)$  is a strain value characterizing swell of a soil specimen at the seating pressure ( $e_0' > e_0$ ),  $r = (e_0 - e_{\underline{\sigma}})/(1 + e_0)$  is a strain value representing a “compressed” part of an oedometer curve ( $e_0 > e_{\underline{\sigma}}$ ),  $e_0$  is in-situ soil void ratio at natural moisture content,  $e_0'$  is void ratio of a swollen specimen at the seating pressure,  $e_{\underline{\sigma}}$  is soil specimen void ratio under applied stress  $\underline{\sigma}$ ,  $\underline{\sigma} = \sigma_{sz} + \sigma_z$  ( $\sigma_z = 0$  when  $\underline{\sigma} \leq \sigma_{sz}$ ).

Equation 1 (or 1a) may be used for the interpretation of oedometer test results. Kodner (1963) and Hansen (1963) showed that the beginning and “tail” of a compression curve may be approximated with a parabola and a hyperbola, respectively. Reznik (2000, 2005) suggested that the intersection point of those parabola and hyperbola may be accepted as the structural pressure value,  $\sigma_{sz}$ . The “tail” of the compression curve may be approximated with the following function

$$\underline{y} = \underline{M}_h + \underline{N}_h \Omega \quad (2)$$

where  $\underline{M}_h$  and  $\underline{N}_h$  are some coefficients,  $\Omega = 1/\underline{\sigma}$ . Here and after, we will use underlined symbols and variable subscripts as it was done in the Author's paper published earlier (Reznik, 2005). Calculation of coefficients  $\underline{M}_h$  and  $\underline{N}_h$  was explained earlier (Reznik, 2005). However, formulas for calculations of those coefficients are included in Appendix 1 for the convenience of readers.

If  $\underline{M}_h$  and  $\underline{N}_h$  are known, it is possible to calculate structural pressure value,  $\sigma_{sz}$ .

When  $\underline{\sigma} = \sigma_{sz}$  and  $\delta = 0$ ,  $(\Delta h_a/h_0)_{total} = r$ .

$$r = \underline{M}_h + \underline{N}_h / \sigma_{sz} \quad (3)$$

$$\sigma_{sz} = \underline{N}_h / (r - \underline{M}_h) \quad (3a)$$

If  $r$  is negligibly small or  $r=0$ , then

$$\sigma_{sz} = -\underline{N}_h / \underline{M}_h \quad (3b)$$

Equations 1 through 3b may be applied to swelling, collapsible, swelling-collapsible, and “ordinary” cohesive soils. Figure 1 illustrates the application of those formulas to interpretation of results of oedometer tests performed on different soils (natural and chemically improved ones). Solid lines in Figure 1 represent experimental (oedometer) curves, and broken lines represent curves obtained during interpretation: coefficients for the calculated curves may be determined using Equations 1 through 3b (when  $\underline{\sigma} \geq \sigma_{sz}$ ).

The following are examples illustrating the application of the proposed method to calculate structural pressure values.

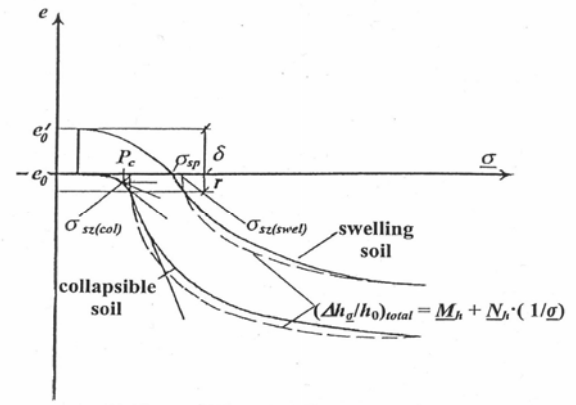


FIG. 1 INTERPRETATION OF RESULTS OF OEDOMETER TESTS PERFORMED ON SWELLING AND COLLAPSIBLE SOILS

### Example 1

Rao and Revanasiddappa (2002) studied the behavior of residually developed red soils occurred in the vicinity of Bangalore, India. The red soils in question are a product of weathering of gneissic parent rock. The void ratio of the above-mentioned soils varies from 0.5 to 1.1. The degree of saturation of *in-situ* Bangalore soils changes from 20% to 70%, and the soils are susceptible to collapse on wetting under a load. Table 1 contains the results of one-dimensional test performed on saturated undisturbed red soil specimen having void ratio  $e_0 = 0.766$ , liquid limit  $W_L = 45\%$ , plasticity index  $I_P = 25\%$ .

The selection of experimental points for calculating quantities of  $\underline{\Sigma}_6$ ,  $\underline{\Sigma}_7$ ,  $\underline{\Sigma}_8$  and  $\underline{\Sigma}_9$  is based on the principle that is illustrated by the data included in Column 6 of Table 1. The numbers included in that column represent the rate of specimen strain with respect to applied stresses. Equation 2 describes a hyperbolic relationship between specimen strains and applied stress values. It does not represent the beginning of a compression curve which describes the elastic properties of soil specimen (Hansen, 1963; Reznik, 1994a, 1994b). The “tail” of a compression curve represents the plastic state of the tested soil specimen. This part of a compression curve is described with a hyperbola. Therefore, when the applied stress decreases, the specimen strain rate must increase. If the strain rate does not increase when the stress values decrease, Equation 2 is not applicable. This fact serves as the criterion for the selection of oedometer curve points which may be included in the calculation process. According to the data contained in Column 6 of Table 1, only three experimental “points” may be used for calculating quantities of  $\underline{\Sigma}_6$ ,  $\underline{\Sigma}_7$ ,  $\underline{\Sigma}_8$  and  $\underline{\Sigma}_9$ . It

has to be noted that the stress – strain information represented by those three “points” is the minimum information required for the performance of that calculation procedure. If the number of “points” is less than three, the test has to be continued. It can be seen that the change of strain rate values has occurred between the stress values of  $1.0 \cdot 10^2$  kPa and  $2.0 \cdot 10^2$  kPa. The number of “points” is three.

Coefficients  $\underline{M}_h$  and  $\underline{N}_h$  were calculated using equations 6 through 6e (Appendix 1).  $\underline{M}_h = 19.57 \cdot 10^{-2}$ ;  $\underline{N}_h = -26.63$  kPa.

The value of  $r$  is a convenient parameter:  $r$  represents the difference between strains of “dry” and inundated specimens subjected to identical stresses. Many researchers in Eastern Europe and CIS (former USSR) accept  $r=0.01$  as an adequate value for the identification of soil collapsibility. The stress value corresponding to  $r=0.01$  is called initial collapse pressure. This is the reason why  $\sigma_{sz}$  is determined for  $r=0$  and  $r=0.01$ . In this example, when  $r=0$ ,  $\sigma_{sz} = 1.36 \cdot 10^2$  kPa, when  $r=0.01$ ,  $\sigma_{sz} = 1.43 \cdot 10^2$  kPa.

TABLE 1 INTERPRETATION OF RESULTS OF OEDOMETER TEST PERFORMED ON A RESIDUAL SOIL SPECIMEN (RAO & REVANASIDDAPPA, 2002, FIG. 2)

$e_s$	$\sigma$ (kPa) $\times 10^2$	$\Omega = 1/\sigma$ (1/kPa) $\times 10^{-2}$	$y = (\epsilon_s^i - \epsilon_s^f)/(1 + \epsilon_s^f)$ or $y = (\epsilon_s^i - \epsilon_s^f)/(1 + \epsilon_s^f) \times 10^{-2}$	$\Omega^2$ , (1/kPa) $^2$ $\times 10^{-4}$	$y \Omega$ , (1/kPa) $\times 10^{-4}$
(1)	(2)	(3)	(4)	(5)	(6)
0.759	0.25	4.00	0.7896	16.00	3.1585
0.741	0.50	2.00	1.8048	4.00	3.6097
0.728	1.00	1.00	2.5381	1.00	2.5381
0.655	2.00	0.500	6.6554	0.2500	3.3277
0.565	4.00	0.250	11.7315	0.0625	2.9329
0.471	8.00	0.125	17.0333	0.0156	2.1292
$n=3$		$\Sigma \Omega = 0.875$	$\Sigma y = 35.4202$	$\Sigma \Omega^2 = 0.3281$	$\Sigma y \Omega = 8.3898$

Notes:

- The upper formula in Column 4 must be applied to swelling and swelling-collapsible soils, the lower formula in Column 4 – to non-swelling and collapsible soils.
- The oedometer test was performed on undisturbed water saturated specimen.

### Example 2

Huergo and Crespo (1988) presented results of one-dimensional tests performed on Belgian loessial soils deposited at the south-eastern part of the country. The samples were recovered from 10-meter long and 5-

meter deep trench. The compression curve of the sample recovered from the depth of 0.3 – 0.6 meters (B-soil horizon, layer 9 – according to the Authors' designations) was scaled (Fig. 4, curve a, Huergo and Crespo's paper, 1988) and calculation results were included in Table 2. Natural moisture content of the specimen was 19%, void ratio = 0.662, specific gravity = 2.60, liquid and plastic limits = 34% and 20%, respectively. According to the data contained in Column 6 of Table 2, five experimental points may be used for calculating quantities of  $\Sigma \Omega$ ,  $\Sigma y$ ,  $\Sigma \Omega^2$  and  $\Sigma y \Omega$ . Coefficients  $\underline{M}_h$  and  $\underline{N}_h$  were calculated in the same way as in Example 1.  $\underline{M}_h = 8.22 \cdot 10^{-2}$ ;  $\underline{N}_h = -4.09$  kPa.

TABLE 2 INTERPRETATION OF RESULTS OF OEDOMETER TEST PERFORMED ON A SOIL SPECIMEN RECOVERED FROM A LOESSIAL SOIL (HUERGO AND CRESPO, 1988, FIG. 4, CURVE 2)

$e_s$	$\sigma$ (kPa) $\times 10^2$	$\Omega = 1/\sigma$ (1/kPa) $\times 10^{-2}$	$y = (\epsilon_s^i - \epsilon_s^f)/(1 + \epsilon_s^f)$ or $y = (\epsilon_s^i - \epsilon_s^f)/(1 + \epsilon_s^f) \times 10^{-2}$	$\Omega^2$ , (1/kPa) $^2$ $\times 10^{-4}$	$y \Omega$ , (1/kPa) $\times 10^{-4}$
(1)	(2)	(3)	(4)	(5)	(6)
0.654	0.25	4.000	0.4813	16.0000	1.9254
0.641	0.50	2.000	1.2635	4.0000	2.5271
0.622	1.00	1.000	2.4067	1.0000	2.4067
0.591	2.00	0.500	4.2720	0.2500	2.1360
0.541	4.00	0.250	7.2804	0.0625	1.8261
0.494	8.00	0.125	10.1083	0.0156	1.2635
$n=5$		$\Sigma \Omega = 3.875$	$\Sigma y = 25.3309$	$\Sigma \Omega^2 = 5.3281$	$\Sigma y \Omega = 10.153$

Note:

The upper formula in Column 4 must be applied to swelling and swelling-collapsible soils, the lower formula in Column 4 must be applied to non-swelling and collapsible soils.

According to Equation 2,  $\lim_{\sigma \rightarrow \infty} y = \underline{M}_h$ . However, the calculated value of  $\underline{M}_h$  is less than strain  $y(\sigma)$  corresponding to the “last” stress value,  $\sigma_l$ , applied to the specimen (specifically, 800 kPa). The observed phenomenon may be explained in the following way. When  $\sigma \geq \sigma_{sz}$ , soil structural bonds start to gradually deteriorate (the  $\sigma_{sz}$  limit manifests the beginning of plastic deformations). However, only a part of soil structural bonds will be destroyed by stresses immediately exceeding  $\sigma_{sz}$ . Therefore, the observed total specimen strains will be less than the “theoretical” ones (or “pure” plastic strains). Nevertheless, the measured strain values will be included in calculations of  $\underline{M}_h$  and  $\underline{N}_h$  affecting values of those coefficients. There are other factors which may affect values of  $\underline{M}_h$

and  $N_h$ . For instance, friction forces acting between the soil specimen surfaces and oedometer parts may also affect magnitudes of observed compression strains. As it was mentioned earlier, the structure of tested soil specimen continuously deteriorates during the test. Also, it may happen that density of destructured soil specimen increases during the test leading to temporary decrease of deterioration of the rate of soil structural resistance to compression within some interval of stresses applied to the soil specimen. As a result, the “tail” of the calculated curve described by Equation 2 may be plotted above the experimental points obtained under final stresses. This problem may be resolved using the following technique. Assume that

$$(\underline{M}_h)_{corrected} = \underline{M}_h + \Delta \quad (4)$$

where  $\Delta = (\Delta h_d/h_0)_{total-l} - (\underline{M}_h + \underline{N}_h/\sigma_l)$ ,  $(\Delta h_d/h_0)_{total-l}$  is the observed specimen strain corresponding to the last stress increment,  $\sigma_l$ .

The corrected value of structural pressure,  $\sigma_{sz(cor)}$ , may be calculated using the following formulas

$$\sigma_{sz(cor)} = \underline{N}_h / [r - (\underline{M}_h)_{corrected}] \quad (5)$$

if  $r > 0$

or

$$\sigma_{sz(cor)} = -\underline{N}_h / (\underline{M}_h)_{corrected} \quad (5a)$$

if  $r = 0$ .

The justification of the proposed correction procedure using Equations 4 through 5a was presented by the Author earlier (Reznik, 2005).

According to Table 2 (column 4), the value of  $(\underline{M}_h)_{corrected} = 10.1083 \cdot 10^{-2}$ .

When  $r=0$ ,  $\sigma_{sz} = 0.40 \cdot 10^2$  kPa, when  $r=0.01$ ,  $\sigma_{sz} = 0.45 \cdot 10^2$  kPa.

### Example 3

Table 3 of this paper includes interpretation of results of an oedometer test performed on the undisturbed soil specimen of swelling black cotton soil (Sridharan et al, 1986, Fig. 2 of Sridharan's paper, block sample P<sub>2</sub>) with initial void ratio of 0.995, initial moisture content of 33.0%, liquid and plastic limits of 108% and 37%, respectively. When the specimen was inundated under seating pressure of  $0.0625 \cdot 10^2$  kPa, its void ratio increased to  $e_0^f = 1.011$ , indicating that the tested soil swelled.

It is well known that mechanical properties of soils depend on soil moisture conditions.

To simplify the explanation of interpretation procedures of oedometer test results, soil specimens with different moisture contents representing the “same” soil will be considered as specimens representing “different” soils (or soils with different physical and mechanical properties). This “standardization” allows treating test results of different soils in the same way. (The selection of input information was explained in Examples 1 and 2.) Determination of swell of a specimen,  $\delta$ , may be done using the upper formula in Column 4 (Table 3). Calculation shows that  $\delta = 0.008$ ,  $\underline{M}_h = 11.49 \cdot 10^{-2}$ ;  $\underline{N}_h = -9.22$  kPa. The specimen deformation  $(\Delta h_d/h_0)_l$  after application of stress  $\underline{\sigma}_l = 800$  kPa was  $12.23 \cdot 10^{-2}$  (the input information was scaled from Fig. 2 of Sridharan's et al paper) exceeds the  $\underline{M}_h$  value. Therefore,  $(\underline{M}_h)_{corrected} = 12.23 \cdot 10^{-2}$ . If  $r=0$ , then (Equation 5a)  $\sigma_{sz(cor)} = 0.81 \cdot 10^2$  kPa; if  $r=0.01$  (Equation 5),  $\sigma_{sz(cor)} = 0.89 \cdot 10^2$  kPa.

TABLE 3 INTERPRETATION OF RESULTS OF OEDOMETER TEST PERFORMED ON A SPECIMEN RECOVERED FROM A SWELLING SOIL (SRIDHARAN ET AL, 1986, FIG. 2)

$e_u$	$\underline{\sigma}_l$ (kPa) $\times 10^2$	$\Omega = 1/\underline{\sigma}_l$ (1/kPa) $\times 10^{-2}$	$y = (\underline{e}_0^f - \underline{e}_s)/(1 + \underline{e}_s)$ or $y = (\underline{e}_0 - \underline{e}_s)/(1 + \underline{e}_s)$ $\times 10^{-2}$	$\Omega^2$ (1/kPa) <sup>2</sup> $\times 10^{-4}$	$y \Omega$ (1/kPa) $\times 10^{-4}$
(1)	(2)	(3)	(4)	(5)	(6)
1.011	0.0625	16,000	0	256.00	0
1.005	0.1250	8,000	0.3008	64.000	2.4064
1.000	0.2500	4,000	0.5514	16.000	2.2055
0.980	0.5000	2,000	1.5539	4.0099	3.1078
0.948	1.0000	1,000	3.1579	1.000	3.1579
0.908	2.0000	0.500	5.1629	0.2500	2.5815
0.849	4.0000	0.250	8.1203	0.0625	2.0301
0.767	8.0000	0.125	12.2306	0.0156	1.5288
$n=4$		$\Sigma_6=1.875$	$\Sigma_7=28.6717$	$\Sigma_8=1.3281$	$\Sigma_9=9.2983$

#### Note:

The upper formula in Column 4 must be applied to swelling and swelling-collapsible soils, the lower formula in Column 4 must be applied to non-swelling and collapsible soils.

### Example 4

Kamruzzaman et al (2009) analyzed the degree of improvement of mechanical properties of cement-treated Singapore marine clay. The Authors presented results of oedometer tests performed on highly plastic soft clay with liquid limit  $W_L=87\%$ , plastic limit  $W_P=35\%$ , total and dry unit weights of  $15.92$  kN/m<sup>3</sup> and  $9.36$  kN/m<sup>3</sup>, respectively. “Apparent pre-consolidation pressure” value (the Authors' definition) of in-situ clay

was 60 kPa. “Apparent pre-consolidation pressure” values shown in Fig. 8 of the above-mentioned paper were determined by the Authors only for four (of six) tested soil specimens representing Singapore marine clay treated with 10, 20, 30 and 50% of cement (Kamruzzaman et al, 2009). No “apparent pre-consolidation pressures” were determined for two specimens which represented the remolded untreated clay and the clay treated with 5% of cement (it is possible that configurations of compression curves characterizing consolidation behavior of those two specimens did not permit the Authors to apply some graphical method that was used for determination of  $P_c$  values in other four cases).

TABLE 4 INTERPRETATION OF RESULTS OF COMPRESSION TEST PERFORMED ON A SAMPLE OF SINGAPORE MARINE CLAY TREATED WITH 30% CEMENT ( $e_0 = 2.66$ ) (AFTER KAMRUZZAMAN ET AL, 2009, FIG. 8)

$e_s$	$\sigma_s$ (kPa) $\times 10^2$	$\Omega = 1/\sigma_s$ (1/kPa) $\times 10^{-2}$	$y = (\frac{\sigma_s^* - \sigma_s}{1 + \sigma_s})$ or $y = (\sigma_s - \sigma_s^*) / (1 + \sigma_s)$ $\times 10^{-2}$	$\Omega^2$ (1/kPa) <sup>2</sup> $\times 10^{-4}$	$y \Omega$ (1/kPa) $\times 10^{-4}$
(1)	(2)	(3)	(4)	(5)	(6)
2.6	0.50	2.00	0.0	4.00	0.0
2.6	1.00	1.00	0.2732	1.00	0.2732
2.6	2.00	0.50	0.5464	0.25	0.2732
2.6	4.00	0.25	1.3661	0.0625	0.3415
2.4	8.00	0.125	4.6448	0.0156	0.5806
2.0	16.00	0.0625	15.5738	0.00391	0.9734
1.6	32.00	0.03125	28.9617	0.00098	0.9050
1.1	64.00	0.01562	41.2583	0.00024	0.6447
$n=3$		$\Sigma e = 0.1093$	$\Sigma y = 85.79$	$\Sigma \Omega = 0.0051$	$\Sigma y\Omega = 2.522$

Note:

The upper formula in Column 4 must be applied to swelling and swelling-collapsible soils, the lower formula in Column 4 must be applied to non-swelling and collapsible soils.

The compression curve obtained for the tested specimen of Singapore marine clay treated with 30% of cement was scaled from Fig. 8 of the referenced paper. The method of selection of input information was explained earlier (Example 1). According to Table 4,  $n=3$ ,  $\underline{M}_h = 47.86 \times 10^{-2}$ ,  $\underline{N}_h = -528.98$  kPa.

Table 5 compares structural pressure values calculated using the interpretation technique offered in this paper with “apparent pre-consolidation pressures” ( $P_c$ ) shown in Fig. 8 (Kamruzzaman et al, 2009). The first two lines of Table 5 do not contain any “apparent pre-consolidation pressure” values determined in the Kamruzzaman et al paper – configurations of

compression curves obtained for the tested untreated (remolded) clay specimen and the specimen treated with 5% of cement did not allow one to apply graphical methods for determination of  $P_c$  values (for instance, the Casagrande’s construction). It was possible to calculate structural pressure values using Equation 2.

The last line of Table 5 excludes the calculated value of structural pressure value (for the clay specimen treated with 50% of cement): the compression curve is “short” – at least one more load step had to be applied to the specimen to obtain necessary information. The analytical method offered in this paper (Example 1) “instructs” the computerized testing device (or helps the person performing oedometer testing) to conduct a technically correct test the most effective way.

TABLE 5 COMPARISON OF “APPARENT PRE-CONSOLIDATION PRESSURES” AND STRUCTURAL PRESSURE VALUES

##	% of cement mixed with clay	$\sigma_{sz}$ kPa, $\times 10^2$	$P_c$ kPa, $\times 10^2$
1	0	0.078	<i>b</i>
2	5	1.38	<i>b</i>
3	10	4.85	2.45
4	20	8.24	5.00
5	30	11.05	9.00
6	50	<i>a</i>	19.00

Note:

1- $P_c$  values were determined by Kamruzzaman et al; *a*-more experimental points are needed, *b*-no data was provided

## Conclusion

The paper presents a method for calculation of structural pressure values using oedometer test results. Simple formulas are suggested for calculations of structural pressure values which are unique properties of each soil.

The rule for the selection of experimental input information that may be used for the above-mentioned calculations is explained, and examples illustrating the application of the proposed method are included in the paper.

The proposed standardized calculation procedures are applicable to interpretation of results of oedometer tests performed on different natural or chemically treated soils. The proposed formulas enable experimenters to select objectively (for calculation purposes) all input information as soon as it is produced by the testing devices. The presented method allows one to control the length of the test

avoiding unexpected problems associated with losing some important experimental data.

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#### Appendix 1

Coefficients  $\underline{M}_h$  and  $\underline{N}_h$  may be found using the following equations (subscripts “i” of calculated quantities  $\underline{\Sigma}$  will be kept the same as it was done earlier (Reznik 2005)

$$\underline{M}_h = (\underline{\Sigma}_s^* \underline{\Sigma}_7 - \underline{\Sigma}_6^* \underline{\Sigma}_9) / (n \underline{\Sigma}_s - \underline{\Sigma}_6^* \underline{\Sigma}_6) \quad (6)$$

$$\underline{N}_h = (n \underline{\Sigma}_9 - \underline{\Sigma}_6^* \underline{\Sigma}_7) / (n \underline{\Sigma}_s - \underline{\Sigma}_6^* \underline{\Sigma}_6) \quad (6a)$$

$$\underline{\Sigma}_6 = \underline{\Sigma} \Omega \quad (6b)$$

$$\underline{\Sigma}_7 = \underline{\Sigma} y_i \quad (6c)$$

$$\underline{\Sigma}_8 = \underline{\Sigma} \Omega^2 \quad (6d)$$

$$\underline{\Sigma}_9 = \underline{\Sigma} \Omega y_i \quad (6e)$$

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- “Interpretation of Potential Fields of Charged Conductive Spheroidal Bodies”, CIS, 1972 (in Russian).
- “Determination of Groundwater Paths Using Method of Streaming Potentials”, 1990.
- “Deformation Zones Under Square Footings Supported by Clayey Soils”, 1998.